# DESIGN OF SCOUR PROTECTION FOR THE BRIDGE PIERS OF THE ØRESUND LINK

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**Abstract**

*The Øresund Link between Denmark and Sweden consists of a cable stayed bridge, a man-made island and an immersed tunnel. The engineering design of the scour protection of the 45 approach bridge piers was made on the basis of numerical and physical modelling. The numerical studies included determination of the hydrographic design conditions and wave/current induced bed shear stresses along the bridge alignment, enabling the selection of the most exposed piers for each bridge segment and pier type. Physical model tests were then performed with the selected piers to investigate the necessary scour protection. This article describes the methods applied and the conclusions and resulting scour protection design.*

## Introduction

The 16 km long fixed link across the straight between Denmark and Sweden, the Øresund Link, will consist of three main elements: 1) an immersed tunnel, 2) a man-made island, and 3) a cable stayed high bridge. The immersed tunnel crosses the eastern channel between Kastrup on the Danish coast and the man-made island Peberholm just south of Saltholm. The cable stayed high bridge, including approach bridges, crosses the main shipping route in Øresund, Flinterenden, and reaches the Swedish coast at Lernacken. The location of the bridge is shown in Figure 1.

This article concerns the design of the scour protection for the approach bridge piers, which has been investigated through both numerical and physical model tests carried out by Danish Hydraulic Institute (DHI).

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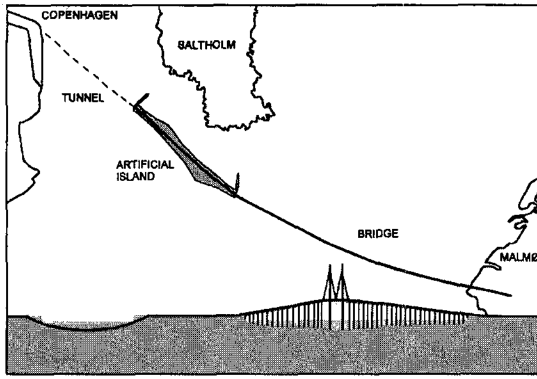


*Figure 1 Alignment of the Øresund Link.*

**Bridge Design**

The approach bridges are founded on 45 concrete piers at water depths less than 10 m. The cross-sections of the piers are prismatic with slightly streamlined up- and down-stream ends as shown in Figure 2.b. The dimensions of the pier shafts at seabed level are approximately 6 m x 18 m, and those of the supporting caissons 6 m x 18 to 22 m.

Due to a strict environmental requirement of zero blocking of the water exchange between the North Sea and the Baltic Sea, it was decided to place the pier caissons in excavated pits in the sea bed, see Figure 2.a. The zero blocking requirement further implied that only a few of the supporting caisson walls were allowed to protrude above the original sea bed level, and that the scour protection had to be placed flush with the sea bed.

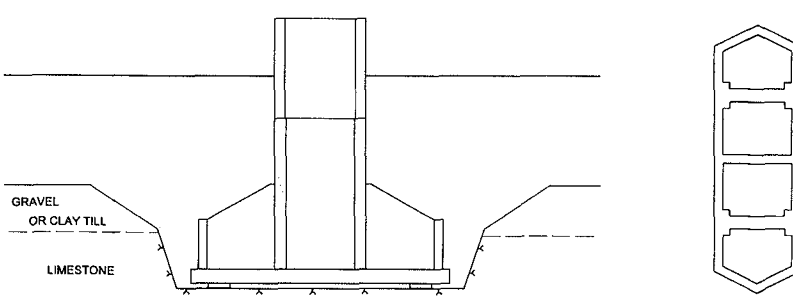


*Figure 2 (a) Vertical cross-section of bridge pier placed in excavation*
*(b) Horizontal cross-section of streamlined pier shaft.*

Placing the caissons in the excavated pits and founding them directly on the hard Copenhagen Limestone further enabled the bridge to withstand the large forces of ice and design ship impacts, but also eliminated the risk of undermining of the caissons, which is usually the major risk of bridge scour.

To ensure the necessary geotechnical stability of the piers, backfilling of the excavated pits was required. The main purpose of the scour protection was thus to contain the backfill inside the excavation.

The geotechnical conditions vary along the bridge alignment, with a seabed consisting of hard non-erodible limestone covered by a layer of clay till or sand/gravel. The thickness of the erodible top layers varies between 0 and 4 m along the bridge alignment. Large differences are even found from one side of the excavation to the other at each pier location.

**Numerical Studies**

The hydrographic conditions in the region has been derived from intensive measuring campaigns and numerical model studies prior to the scour investigations and model testing, and these have been used to define the Design Requirements. From the previous studies and the specifications in the design requirements, the relevant design conditions for waves, currents and water levels along the bridge alignment have been established.

The wave conditions varies along the alignment and are relatively directional. The maximum significant design wave height, $H_s$, is up to 2.7 m for southerly wind directions. The wave heights are generally lower for the near-shore piers due to depth limited breaking of waves. The design wave heights for events with a 100 and 10,000 year return period (yrp) are shown in Table 1 (intervals).

| *Wave heights, Hs (m)* | Northerly | Southerly |
|---|---|---|
| 100 yrp | 1.3-2.0 m | 1.2-2.3 m |
| 10,000 yrp | 1.4-2.6 m | 1.2-2.7 m |

*Table 1 Design Wave Heights.*

The current conditions are nearly bi-directional, with the angle between the main current direction and pier orientation vaired between 10 and 20°. The maximum design current velocity was found to be 3.8 m/s close to the navigation channel for southward currents. The design currents velocities (intervals) are shown in Table 2.

| *Current velocities* | Northward | Southward |
|---|---|---|
| 100 yrp | 1.1-1.8 m/s | 1.2-2.7 m/s |
| 10,000 yrp | 1.4-2.4 m/s | 1.7-3.8 m/s |

*Table 2 Design Current Velocities.*

There is an inherent correlation between extreme wave and current events, as these events are primarily caused by the same meteorological conditions, and it was subsequently decided to use a straight forward combination of wave and current conditions in the design.

As the correlation between water levels and wind directions showed a rather large scatter and since the scour protection is most exposed at low water, assuming equal wave and current conditions, it was decided to adapt a conservative most probable low water level in the design. This level varied between -0.5 m and +0.5 m.

Based on the data described above, the bed shear stress distribution along the bridge alignment was computed using the numerical sediment transport programme STP, which is a part of LITPACK, DHI's deterministic modelling system for littoral processes. All relevant combinations of wave and current directions were calculated using a bed grain size of 0.2 mm. The resulting shear stresses are shown in Figure 3.

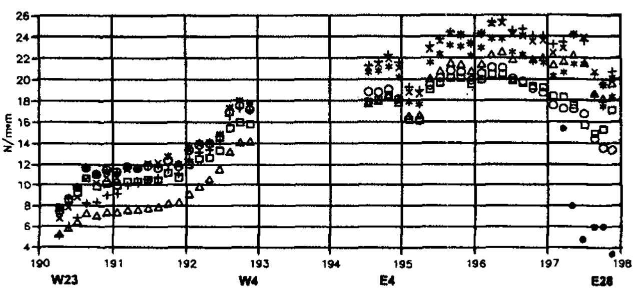


*Figure 3 Bed shear stress along the bridge alignment.*

The limited variation in the size of the maximum shear stress for the various wave directions indicate that the bed shear stress is largely dominated by the current. This is even more clear from the direction of the maximum shear stress, as the waves are only able to deflect the maximum shear stress direction approx. ±10-15° away from the main current direction. This fact was important for the selection of set-up of the physical model and definition of the test programme.

The shear stresscs shown in Figure 3 are not representative of the shear stresses actually experienced by the scour protection once the bridge piers and the protection have been placed, as the piers and the rocky protection themselves alter the flow pattern fundamentally by amplification and by generation of turbulence.

However, the figure enables the selection of the most exposed piers of each pier type to be tested in the hydraulic model along with the corresponding design conditions to be used in the model.

Three types of pier caissons exist, viz. Type A, B and C. These types are characterised by the number of supporting caisson walls and whether these protruded above sea bed/scour protection level. Type A (W11-W4 and E4-E11) have seven supporting walls terminated just below original sea bed level, Type B (W20-W12 and E12-E26) have five supporting walls that protrude up to 1.0 m above original sea bed level, and Type C (W23-W21 and E27-E28) have seven walls that protrude up to 1.0 m above sea bed level.

The piers closest to the Swedish coast were placed in a 100 m wide excavated construction channel. The local bed levels were thus lower than those used for the computation of the bed shear stresses above. Using the actual bed levels significantly reduced the bed shear stresses, as indicated with • marks on Figure 3, and these piers could therefore be designed as the less exposed western piers.

Based on Figure 3, the following piers were selected for further investigation in the physical model:

- E11, representing all Type A piers
- E17, representing the eastern Type B piers
- W12, representing the western Type B piers

No tests were performed on Type C piers, as these were all relatively sheltered.

**Physical Model Tests and Scour Protection Design**

A physical 2D model was constructed to a linear scale of 1:40 in a 5.5 m wide flume in which unidirectional irregular waves and current could be generated simultaneously. The model set-up is shown in Figure 4.

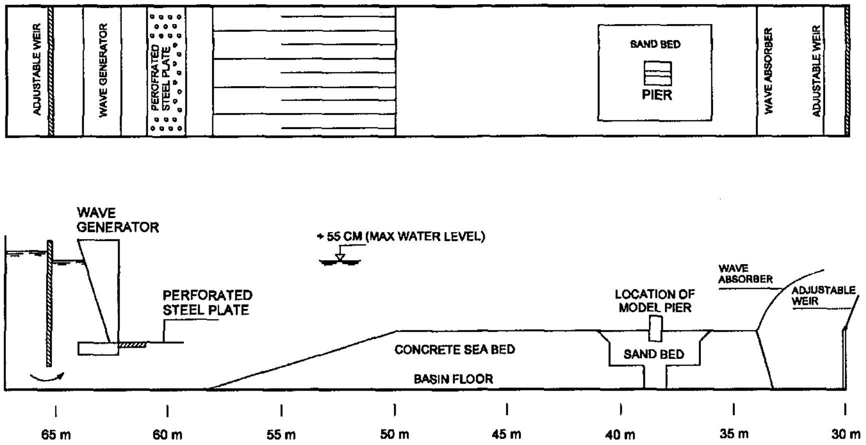


*Figure 4 Two-dimensional model flume set-up.*

The conditions created in the 2D model, combining extreme waves and currents from different directions into one single direction, is estimated to create more critical conditions for the scour protection, as the amplification patterns of shear stresses from waves and currents coincide.

A total of fourteen test series were carried out on three different pier types in order to investigate the stability of the scour protection. Each test series included three standard tests: 1) 100 y waves and current, 2) 10,000 y waves and 100 y current, and 3) 10,000 y waves and current. Each test had a duration corresponding to four hours in nature.

Three different current (and wave) directions relative to the pier were studied. The first tests with current directions of 10° and 40° showed that the damage pattern is rather sensitive to the direction. The tests yielded *acceptable damage* and *collapse* respectively for otherwise identical situations. As the current was found to dominate the scour process outside the protection, it was concluded that a current direction of 20° relative to the pier should be applied in the design.

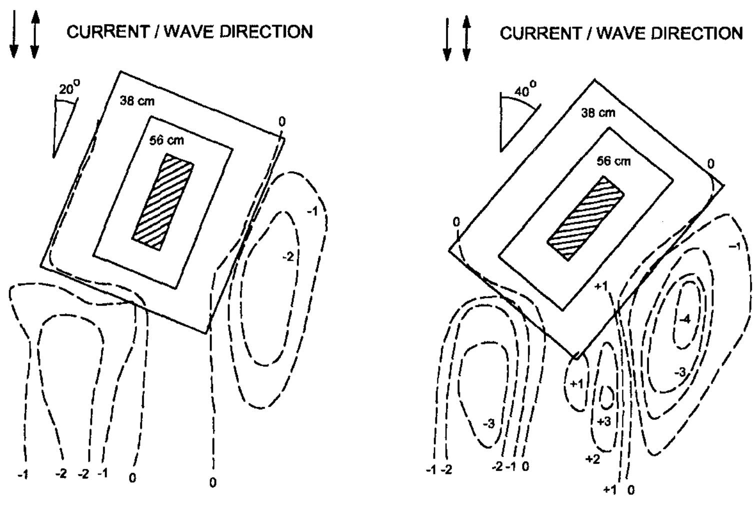


*Figure 5 Typical damage pattern for 20° and 40° pier alignment.*

Two types of damage were observed, viz. direct and indirect damage. Direct damage is caused by the direct action of waves and current on the surface of the protection, whereas indirect damage at the edges of the protection is a result of the scouring of the seabed just outside the protection resulting in stones being moved into the down-stream scour holes by wave action. Figure 5 shows some typical patterns of scour outside the protection and the resulting damage at the edges.

Indirect damage occurred along the downstream side and downstream end of the protection. The direct damage occurred in three areas: At the downstream side, the downstream end, and in the upstream cells (between the supporting walls) or all cells along the pier. Figure 6 shows the areas where damage occurred for a successful test.

To prevent any damage from impacting stones on the concrete, it was specified to use the largest stones of each fraction in the upstream cells and to place these with great caution. In the down stream area the stone size was slightly increased to reduce the number of stones being moved.

As a consequence of the results of the first test series showing larger damage than originally estimated, the test programme was currently changed and very often the layout of the scour protection in a test series was defined from the results of the previous test series in order to optimise the design.

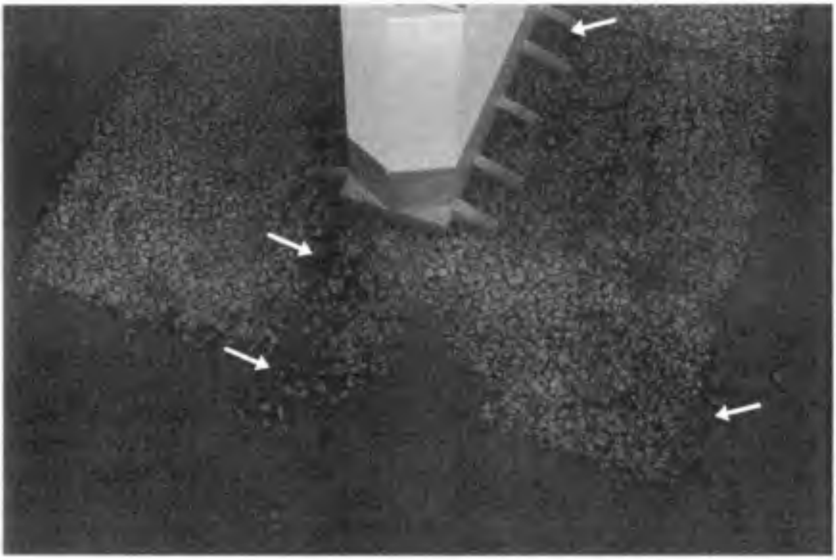

*Figure 6 Typical damages observed in the physical model (looking upstream).*

In addition to the traditional stability tests, it was decided to carry out a tests to investigate the effects of a thin layer of erodible material on top of the limestone. This test showed that the extent of the protection could be reduced significantly for piers placed in areas where the limestone is only covered by a thin layer of erodible soil. Figure 7 shows a photo taken after Test 3 (10,000 yrp). The dark areas is the limestone surface being exposed where the erodible material has been washed away.

Finally a test was carried out to investigate the time scale for the scouring in sand/gravel and clay till outside the protection. Analysis of the results indicated that the scour outside the protection would develop rapidly if the sea bed consisted of sand, whereas it would not exceed 1.0 m within an average 10 year period, if the sea bed consisted of 30 mm gravel. For clay till the scour was found to be in the order of 0.4 m for an average 10 year period, but this value could vary between 0.1 m and 3-4 m corresponding to fully developed scour holes. It is important to note that the values above are determined assuming homogeneous conditions. In reality the sea bed consists of a mixture of sand, gravel, clay till and boulders. The sea bed will therefore to a certain extend tend to be self-armouring if erosion occurs.

The uncertainties of estimating the parameters for the clay till were considered too large to facilitate in the design, but indicated that it might be possible to include this in the design if more intensive studies and possibly a comparison of the results with available field observations were carried out.

*Figure 7 Results of test with shallow limestone (looking upstream).*

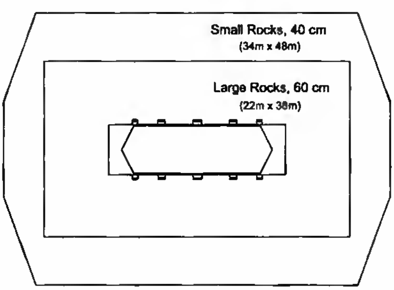


*Figure 8 Plan layout of the largest scour protection (Pier Type A).*

The final layouts of the scour protection were derived by comparing the damage patterns of each test series with the design requirements and taking the local soil conditions into consideration. An example of the protection layout is shown in Figure 8 and Figure 9.

It has further been specified in the design that the protection shall be adjusted according to the actual conditions found on site in the construction phase.

The most important change in the design concerned the back fill. It was originally foreseen that the caissons were to be back filled with the sound friction materials recovered during excavation of the pits, and to place a filter layer between the back fill and the scour protection. However, the large difficulties experienced by the contractor keeping the back fill in place while placing the filter layer, caused him to substitute the native back fill with imported, coarse quarry run. This change in the original design has further increased the safety of the scour protection, and in case the protection is damaged, the maintenance costs will reduced significantly.

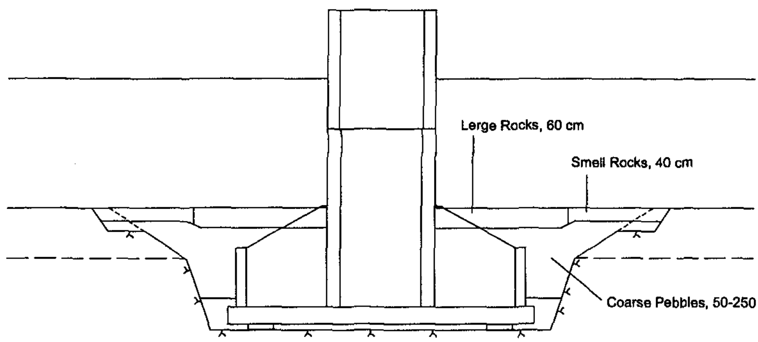


*Figure 9 Cross-section of scour protection around Type A piers.*

## Conclusion

The detailed hydrographic design data were used in the design of the scour protection, including numerical calculations and hydraulic model tests. The test programme was continuously changed to optimise the design. This resulted in a differentiated design of the scour protection along the bridge alignment that fulfils the given requirements, taking both the pier specific hydrographic, geometrical and geotechnical conditions into account.

## References

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/2/ DHI. Hebsgaard, M. et. al. (1997). Model tests with Scour Protection of Bridge Piers, Øresund Link. Clients property.